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# WANETA HYDROELECTRIC PROJECT

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POWER DIVISION

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# WANETA HYDROELECTRIC PROJECT

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## SYNOPSIS

The problems arising in locating, designing and constructing a run-of-river hydroelectric project are described in this paper. Though there were no serious difficulties encountered as to foundation conditions, nor any unusual design problems due to the size of the project, there are problems of interest to the planner of a dam in a deep winding gorge with a swift river subject to large periodic variations in flow.

A review of a model study is included, as well as the description of a novel and successful method of sluice closure.

## General

The Waneta Power Plant, located at the mouth of the Pend d'Oreille River, a quarter mile north of the International Boundary in British Columbia, is a hydroelectric development constructed for the Consolidated Mining and Smelting Company of Canada, Limited (Cominco). The sole function of the project is the generation of electricity. The dam, located in a narrow canyon, is of the straight gravity type with a design height of 219 feet and a length of 950 feet. The reservoir, about 4 miles long, has a total storage of 25,000 acre-feet, about 8,000 of which are usable for power. Two 120,000 hp units are installed in the power house at present. Space is available for the installation of two additional units.

The Pend d'Oreille River drains portions of Montana, Idaho, Washington and British Columbia. The early explorers were disappointed in finding that the river was too full of rapids to become a trade route, but these rapids are now the sources of power. The development of the Pend d'Oreille River and its tributaries in the United States has already begun, in the form of the Hungry Horse, Cabinet Gorge, Albeni Falls and the Box Canyon projects.

Power demands of Cominco for use in the reduction of zinc and lead ores and for the manufacture of nitrogenous fertilizer made desirable immediate development of some of the potential in the 16 mile reach of the Pend d'Oreille in Canada. About 400 feet of head are available in this reach, at least 200 feet of which could readily be developed by a dam close to the Columbia River.

## Selection of Site

Eventual development of all the power potentially available in the Pend d'Oreille required that the dam should be located as closely as possible to the Columbia River. The river flows into the Columbia over partially exposed ledges creating a drop of about 10 feet, depending upon the stage of the

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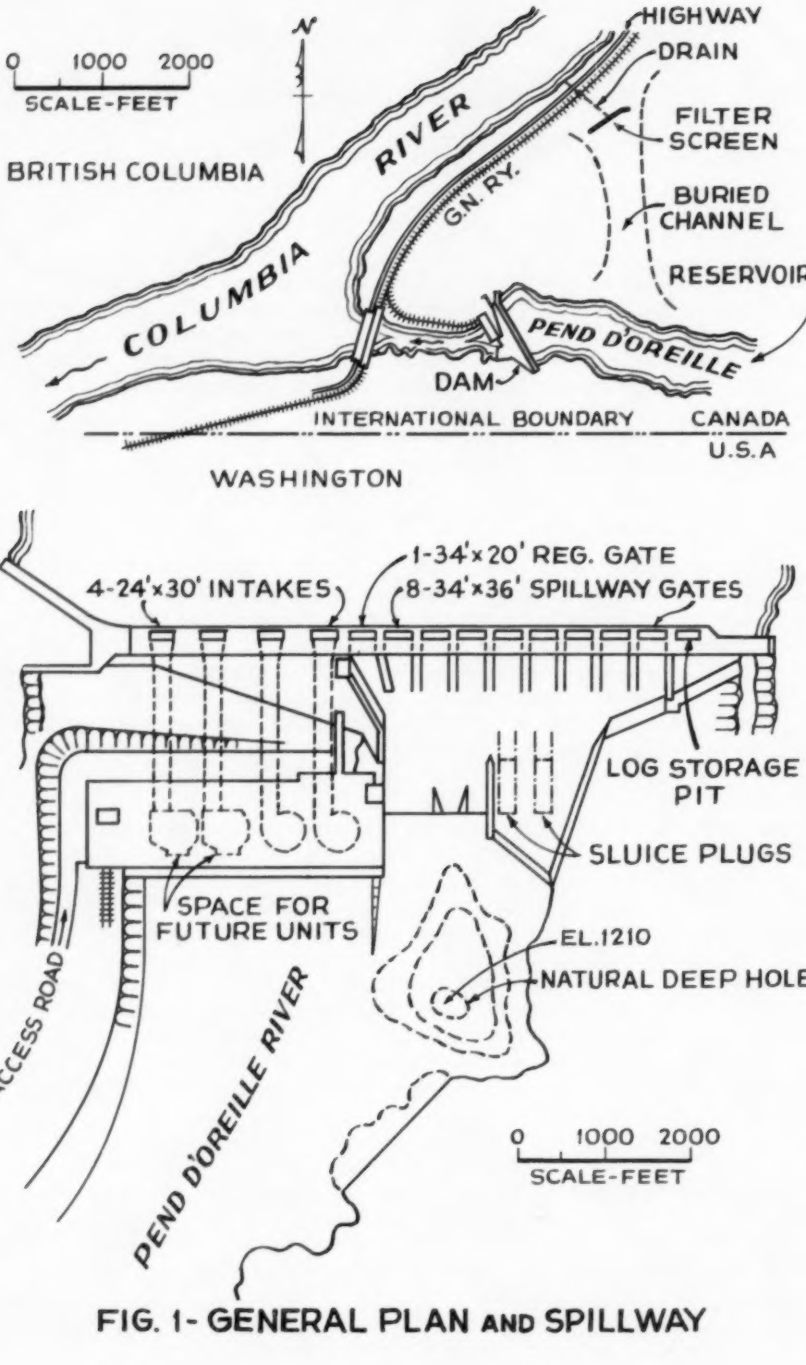



FIG. 1- GENERAL PLAN AND SPILLWAY

Columbia. Some of this drop could economically be made available for power by cutting a channel at this point, provided that the lower water level thus created became the tailwater of a plant located at the rapids, which are about 1,000 feet upstream from the river's mouth. Borings indicated a favorable dam site directly upon the hard rock forming these rapids. At this point, the canyon narrows considerably so that the river bed is about 150 feet wide. The profile across the stream shows steep banks, consisting of exposed rock, rising to about half the height of the dam, sloping more gently thereafter just sufficiently to provide adequate length at El. 1521 for the required dam equipment.

A suitable power house location was not so obvious. A building, about 340 feet in length as required to house four large units, could not be located at the foot of the main dam. Furthermore, virtually the full length of the main dam was required for spillway purposes. Consideration was given to a downstream power house location, assuming diversion through penstock tunnels. On the basis of a maximum flood of 125,000 cfs during diversion, it was computed that two 50 feet diameter partly lined tunnels, 750 feet long, would be required. The cost of such a diversion method being prohibitive, it was decided to use the existing or widened river bed for diversion and to design the penstocks and tunnels solely for power requirements.

Since power was required at the earliest possible date, it was decided to separate the power house construction from the dam construction by confining the power house work within an independent cofferdam which would not be topped by any anticipated flood. Final power house location was determined, keeping in mind the advantage for this project of a cofferdam independent of the dam cofferdam; the advantage of penstocks of minimum length, with favorable radii on the fewest possible number of bends; and the necessity for a site which would not interfere with the dam foundation.

The site chosen was originally a rock bluff overlooking the river, reaching to a height of 150 feet. This location permitted use of a rim of rock around the foot of the bluff as a natural cofferdam within which the excavation for and construction of the power house were carried on without interruption during the flood periods. The total rock excavation for the structure above draft tube deck level, however, was over twice the cubage of the power house superstructure, an amount justified by the saving of an expensive cofferdam and by the saving of the cost of longer penstocks.

The adopted location for the power house creates a tailbay which is bypassed by the main river flow. See Fig. 1. Discharge from the turbine draft tubes is directly downstream.

## Selection of Units and Head

Ultimate development of the remaining power potential of the Pend d'Oreille River in Canada would probably be at a favorable dam site known to exist several miles upstream from the Waneta dam site. This circumstance led to the proposal to fix the normal water surface of the Waneta reservoir at El. 1516, which during the design flood would increase to El. 1520.

Normal tailwater elevation for the Waneta Plant was based upon the regulated average low flow of the Pend d'Oreille during a median year. This flow, occurring generally from August to November, is about 9,700 cfs. Because of channel improvements downstream of the dam, this flow corresponds to a tailbay elevation of 1303. For five months of the year, the Columbia River is sufficiently high to increase this tailbay level, since the power plant is located

close to the Columbia. The two streams are in flood at about the same time of the year.

With a normal available gross head of 213 feet, the units are designed for a net head of 210 feet, capable of developing 120,000 hp at a speed of 120 rpm and a discharge of about 5,700 cfs. Two units were purchased for initial installation. During a selected median year, the capacity factors, based on 120,000 hp rating of one, two, three and four units are 0.97, 0.94, 0.88 and 0.78, respectively. Generation by four units would be 2,460,000,000 kwhr per year.

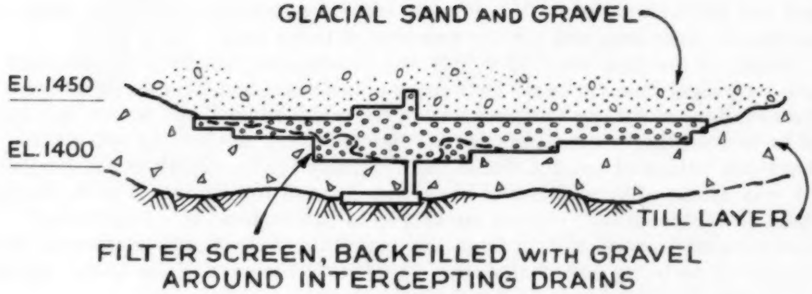
## Buried River Channel

Investigations of the reservoir site disclosed the existence of a buried channel in rock, used by an ancient river. See Fig. 1. This channel extended from a point on the north bank of the reservoir, about 1,500 feet upstream from the dam, to a point on the east bank of the Columbia about a half mile north of the reservoir. As revealed by borings, the rock channel was roughly 500 feet wide, with its bottom about 100 feet below normal reservoir level. This channel was filled to a depth of over 300 feet with glacial till and overlying alluvial sand, gravel and boulders.

At the north end of this natural fill, a sand bank at its angle of repose, about 300 feet high, overlooks the Columbia River, a highway and a railway. At the foot of the bank below the buried channel, a maintenance problem has existed for years because of springs issuing from the foot of the slope. Presumably, these springs drain the buried channel which slopes toward the Columbia River.

Concern was felt by all parties when it was realized that an additional 100 foot head of water from the reservoir at the upper end of the buried channel might increase the flow of the springs by percolation water. The services of Dr. Karl Terzaghi of Harvard University were engaged to consider what remedial measures could be taken to prevent any trouble at the foot of the sand slope caused by increased seepage. Under his direction, a system of drains designed to intercept and divert percolation water was constructed, in consequence of which it is anticipated that maintenance at the toe of the sand slope will be much less than has been required during previous years.

The drainage system (see Fig. 2), as finally constructed, consists essentially of a tunnel and filter screen. The tunnel is 5 feet by 7 feet in section and 375 feet long. It extends inward from the toe of slope and ascends along the bottom of the buried channel, being partly in rock and partly in the overlying impervious glacial till. A 24 inch concrete pipe draining the filter screen is installed in the tunnel. The filter screen was constructed by removing all material in a space bounded by two parallel vertical planes about 4 feet apart, extending at right angles across the buried channel. The removed material was replaced with graded gravel and tile drains. The screen has the irregular impervious bottom of the buried channel as its lower boundary and extends upward to El. 1440. The maximum limits of the screen are approximately 40 feet in height and 400 feet in length across the channel. It is believed that all percolation water coming from the reservoir will be intercepted by the screen and diverted harmlessly into the Columbia River via the 24 inch drain. Six months after filling the reservoir, there was no evidence in observation wells that any water from the reservoir had advanced down the buried channel to the filter screen.

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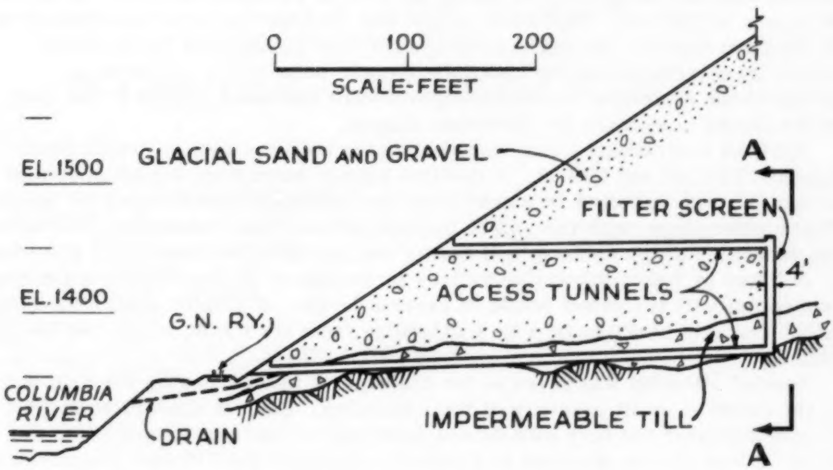


SECTION AT LOWER END OF BURIED CHANNEL

FIG. 2 FILTER SCREEN AND DRAIN

## Dam

The dam is a concrete gravity structure which is straight except for a slight curve at the north end. The maximum design head is 219 feet. Fig. 3 shows the river bed block of maximum section. The foundation of the dam is on solid rock which was sealed by pressure grouting. Except at the south end of the dam where a depth of 25 feet of overburden consisting of glacial till, sand and gravel was found, the only excavation required was for the upstream key trench, benching, and for the removal of loose rock.

Design of the dam was based upon the assumption that vertical stresses have a straight line variation between the upstream and downstream faces. The resultant of forces was kept within the middle third, and where this could not be accomplished near the crest of the spillway, reinforcing was added where low values of tension would theoretically occur. Uplift at the foundation was assumed to be equal to reservoir head at the upstream face, diminishing downstream of the grout curtain to tailwater head at a longitudinal drain located on rock 30 feet from the upstream face. Uplift pressures were considered as being distributed over two-thirds the area of the base. Ice load on the dam was assumed at 12,000 lb per lin ft.

The weight of concrete was assumed at 140 lb per cu ft. The basic shape of the dam section is a triangle, with a vertical upstream face and a downstream face sloped at 10 on 7, changing to 10 on 10 below the point where the thickness is 128 feet. This basic shape was modified to form the crest curve of the discharge jet, the deck of the nonoverflow blocks, and the spillway bucket as determined by the hydraulic model. None of the stresses approached the allowable for the strength of concrete used, except at the toes of the blocks containing the diversion sluices.

Vertical contraction joints were left in the spillway dam to create block lengths of 34, 42 and 50 feet. A vertical rubber waterstop, dumbbell shaped in section, was installed 18 inches from the upstream face between all blocks. These waterstops, with vulcanized splices, extend from foundation rock up to the underside of the spillway sill beams and to just below deck level elsewhere.

A 5 foot by 7 foot inspection gallery is located in the lower part of the dam, connecting with the power house at various levels. A gallery containing electrical conduits is shown on Fig. 4 extending from the power house into the intake dam.

Special attention was given to the drainage of water entering the dam and to the relief of uplift pressure at the foundation. Each horizontal pour joint is provided near the face with drains designed to intercept seepage into the joint. These drains are open to vertical contraction joint drains downstream of the waterstops. The pour joints of the penstock intake transitions are similarly protected because of the proximity of the transition to the downstream face of dam. To cut off percolation from upstream along the foundation, an 8 inch square drain was formed on rock 30 feet from the upstream face of dam. To assist in keying the dam to rock, as well as to provide a longer path for percolation, a trench was cut into the foundation at the upstream heel, varying from 3 to 10 feet in depth. A 24 inch drain intercepting the vertical joint drains is laid in this trench, open to tailwater through 12 inch transverse drains, two of which are located in each spillway block. Sidehill blocks of the dam are similarly drained, with transverse drains open to the downstream apron on rock.

Although the grout curtain is designed to prevent, as far as possible, percolation through rock crevices under the dam, it was considered desirable to

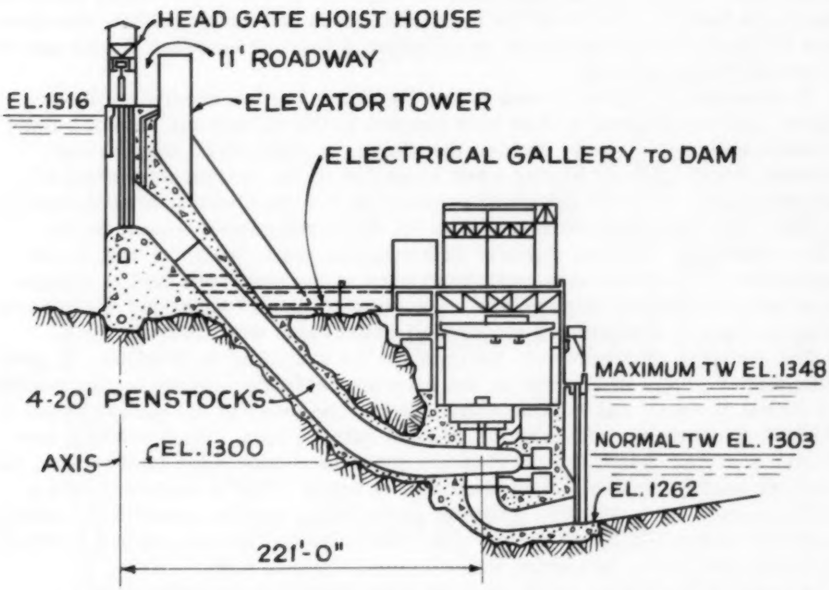


FIG.4-SECTION AT INTAKE AND POWER HOUSE

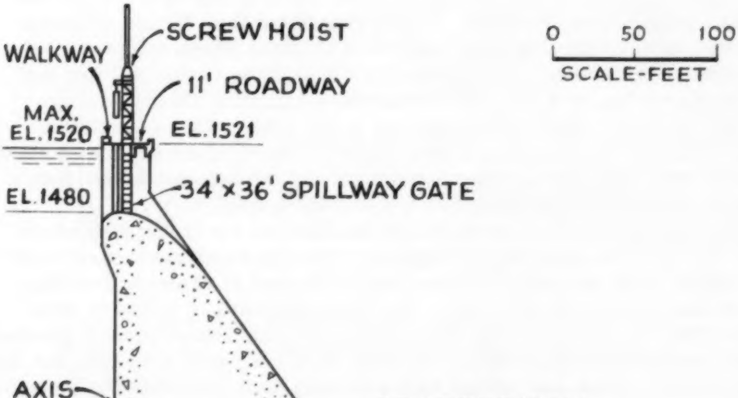


FIG.3-SECTION AT SPILLWAY

provide relief for the uplift pressure due to any percolation which might occur. Accordingly, pillars of porous concrete were formed at points along contraction joints between blocks in the river channel area. These pillars rise from rock to meet transverse drains to tailwater if these transverse drains are not formed directly on rock.

The two sluices (Figs. 5 and 6) are located in the two central spillway blocks, and are unusually wide with respect to the 42 foot and 50 foot blocks in which they are located. Before the sluice openings were plugged and grouted, these spillway blocks were subjected to the overturning effect of a full reservoir. Careful design of reinforcing for the sluice sidewalls was required. The conservative assumption for design of reinforcement in the sluice sidewalls was that transfer of horizontal shear from the dam to the foundation rock varied uniformly from zero at the upstream heel to a maximum value at the downstream toe. Maximum values of shear for the purpose of designing the downstream sluice wall reinforcing were thus obtained.

The strength of concrete in the dam varies according to location. In general, richer mixes were used on the exterior surfaces exposed to the weather, the action of frost, and high velocity water. The mass of the spillway dam is of 2,000 lb concrete, with a water/cement ratio of 0.65. The upstream face of dam above El. 1450, the crest of the dam, the downstream face of dam, the roadway surface and the intake dam blocks are of 3,000 lb concrete with a water/cement ratio of 0.50. Spillway piers, being heavily reinforced concrete members subject to the action of water and freezing, are of nominal 3,000 lb concrete, but with a maximum water/cement ratio of 0.45.

A total of 313,700 cu yd of concrete were poured to form the dam.

## Intakes

The intakes (see Fig. 4) are arranged to the north of the spillway as required by the power house location. Water enters the four 20 foot diameter penstocks through rectangular gate controlled orifices which are protected by trash racks. The average water velocity through the racks is 7 fps, and in the penstocks 18 fps at 5,700 cfs, the maximum turbine discharge. To avoid surface vortexes, gate sills were set at El. 1445. With a maximum drawdown of ten feet during generation, the top of each intake orifice is about 30 feet below reservoir level. The designs for the intakes and transitions to the penstocks were tested by means of a hydraulic model.

The difference in elevation between the intakes and the turbines made it necessary to carry the penstocks in tunnels. The minimum rock cover over the lower end of each penstock is more than sufficient to resist hydrostatic pressure at the spiral case elevation. The penstocks are of 3/4 inch steel plate, back-filled on the outside with concrete, with shrinkage spaces grouted to transmit the hydrostatic pressure to rock. In the adopted arrangement, the intake transitions, which are vented with a 24 inch pipe, turn down to meet the penstock liners on a slope of 12 on 10 1/2. Each penstock is in a vertical plane, turning through a reducing bend of 100 foot radius to meet the 17 foot-6 inch diameter spiral case entrances at El. 1300. The two northerly penstocks are temporarily closed at their lower ends, using 1 1/8 inch thick dished heads.

Each penstock is controlled by a 24 foot by 30 foot fixed wheel riveted headgate. Each gate is operated by means of a 120 ton wire rope hoist, which is supported on a steel tower about 40 feet high, to permit gate maintenance above deck level. A feature of each hoist is a centrifugal blower fan for

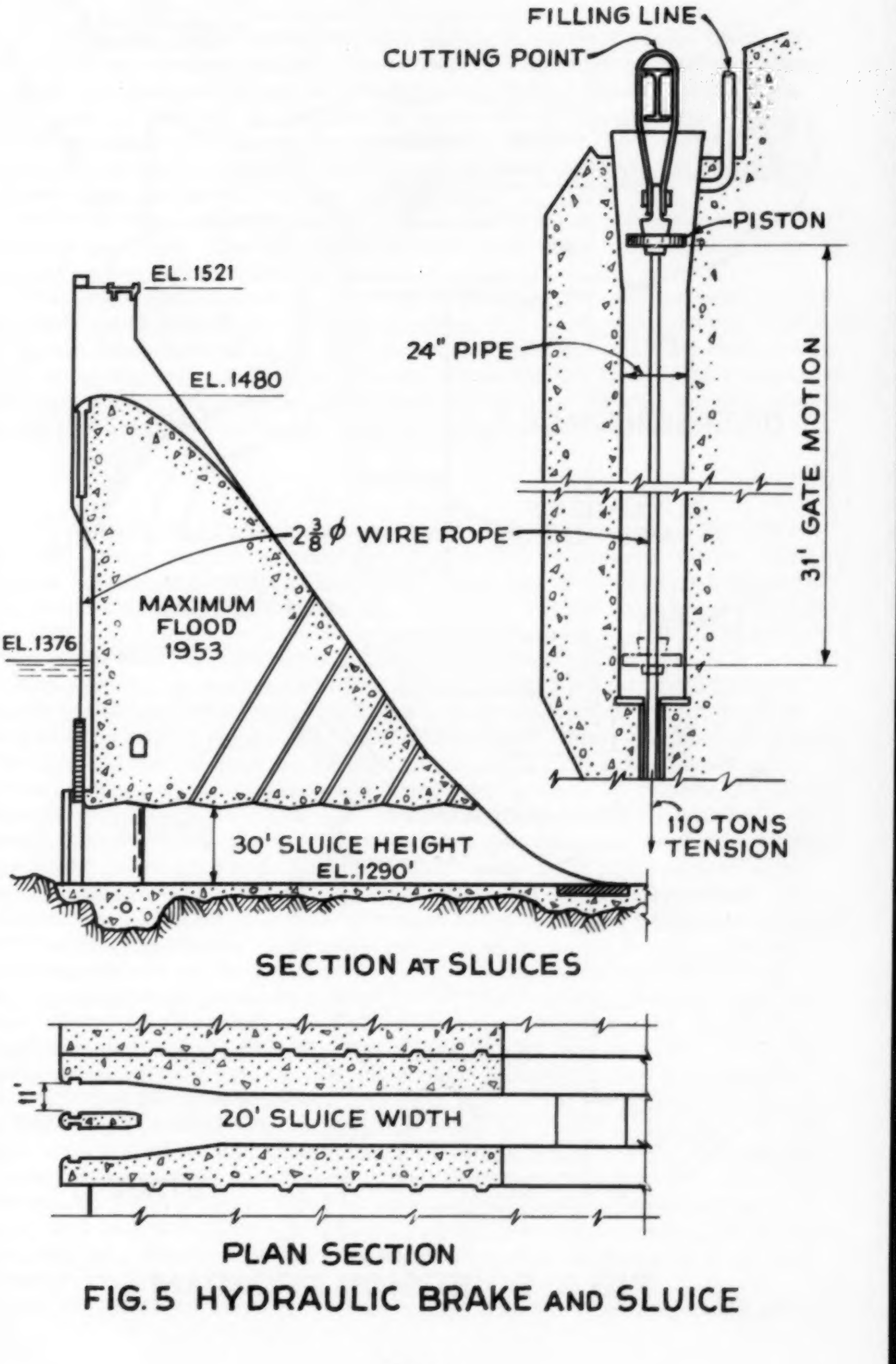


FIG. 5 HYDRAULIC BRAKE AND SLUICE

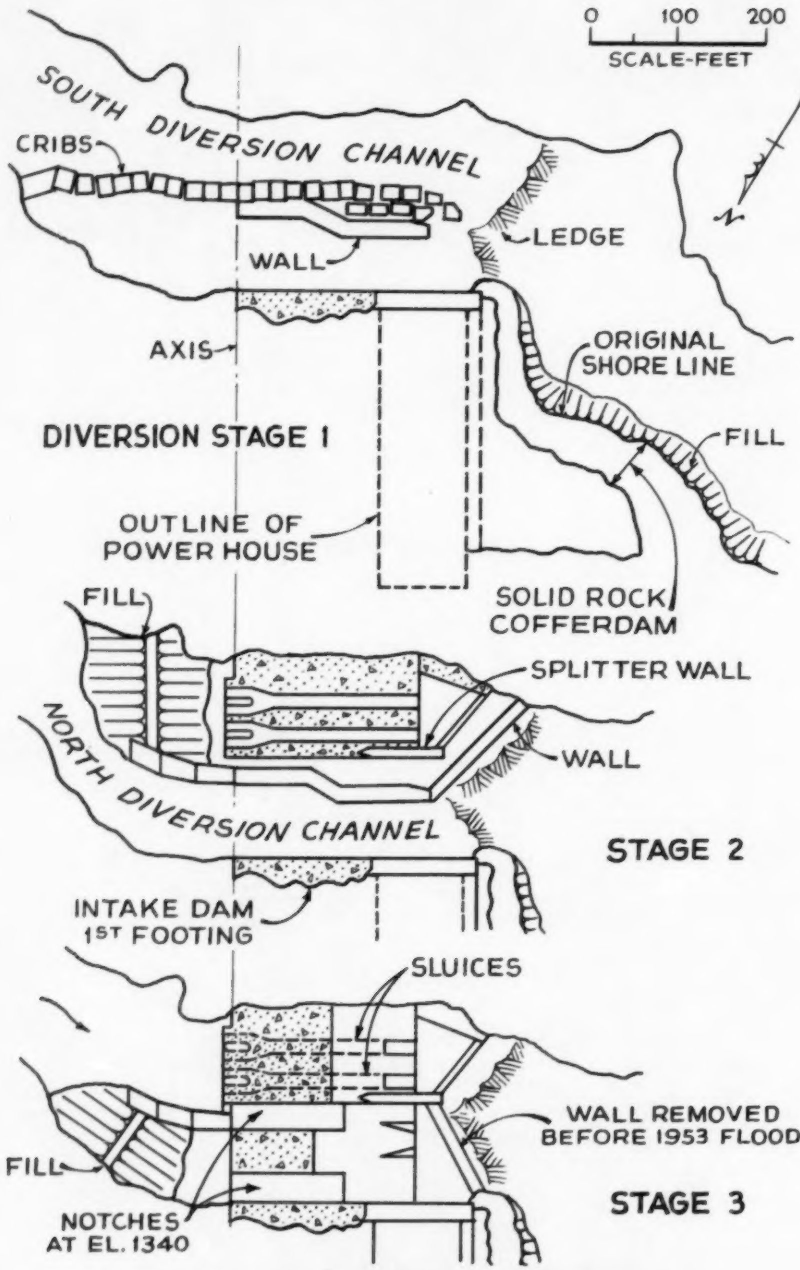


FIG.6-DIVERSION PROGRAM

braking, mounted directly on the motor shaft. The braking effect of this fan is such that motion in the lowering direction, which is accomplished without power, is limited to 2 1/4 times the hoisting speed.

Trash racks are in removable units sliding in vertical guides 3 feet upstream of the headgate guides. The penstocks may be unwatered by the use of stop logs designed for use in the trash rack guides after removal of the racks. Racks and logs are handled by means of a 25 ton overhead traveling crane operating on a runway supported by the headgate hoist towers. Storage space for all logs is provided immediately below deck level in the otherwise unused upper portions of the rack guides.

Trash is removed from the racks by means of a rake operated by the 25 ton stop log crane. The rake is lifted clear of the deck and the trash is deposited on a car at the deck level.

The steel superstructure on the dam is arranged for compactness, permitting gate maintenance, log and rack handling, and trash raking with a minimum of special equipment. The 11 foot roadway on the dam clears the downstream side of the superstructure. An elevator, suited for the handling of passengers and light equipment, provides convenient access from the dam gallery system to the top of the dam and to the upper levels of the power house.

## Spillway

The greatest flood on record in the Pend d'Oreille River was 190,000 cfs in 1948. It was considered desirable to adopt about the same spillway capacity as furnished upstream at the Albeni Falls dam in Idaho, or 300,000 cfs. The Waneta spillway, as constructed, will carry 280,000 cfs without difficulty, the water level in the reservoir rising from normal level at El. 1516 to flood level at El. 1520. Four units would discharge about 20,000 cfs.

The spillway is equipped with fixed wheel gates, operated by individual screw hoists with twin rising stems. There are eight main gates, each 34 feet wide by 36 feet high, and one regulating gate, 34 feet wide by 20 feet high. The type of gate and hoist which were selected have proved satisfactory in severe winters at other Canadian hydroelectric plants. To relieve the partial vacuum which might be created on the downstream face of any gate by accidental discharge over the top, two vertical 8 inch air pipes, open at each end, are built into the gate leaf. Since the upper ends of the pipes project above any probable overflow, air will be supplied beneath the overflow nappe. Bottom seals are formed by means of contact of the planed edge of the skin plate with the steel sill beam. Side seals are formed by 2 inch diameter bronze rods suspended near each pier from the upstream face of gate so as to be held by hydrostatic pressure against the clearance gap between the gate and pier. While all gates may be operated locally, it is possible to operate the regulating gate and the four adjacent main gates by remote control from the power house. These gates are equipped with remote position indicating devices. Although major floods do not occur in winter, it is imperative to have at least these remotely controlled gates ready for use at all times. The side seal seats and wheel tracks are therefore supplied with resistance heaters to prevent seizure of the gates by ice. A bubbler system is provided which discharges air from small orifices in pipes attached to the upstream face of the gate. The action of the bubbler in drawing warmer water from below the surface acts as a deterrent to the formation of ice close to the gates which are, however, designed to resist ice pressure.

The design of the spillway was correlated with the hydraulic model study.

To minimize the number of spillway gates discharging over the rock slope of the south bank of the canyon, the regulating gate was located upstream of the power house.

Fig. 1 shows the general arrangement, which provides an 11 foot service roadway downstream of the gates, and a walkway upstream. The individual screw hoists are supported on a series of bridges which are covered with grating to provide a continuous service walkway. Steel towers, of a height sufficient to permit the screws to raise the gate bottoms one foot above deck level, support the hoist bridges and the guides to steady the gates in service position.

Stop logs are provided for the spillway gate openings and are stored in a well in the south nonoverflow dam. The logs are handled by a 20 ton overhead monorail hoist operating on a continuous track upstream of the hoist bridge towers.

Each pier is of heavily reinforced concrete, designed to resist hydraulic, dead, roadway H-20, temperature and ice loads. Since the bridge would, in the future, be subject to unusual loads during the possible replacement of a spillway gate, stresses due to heavy concentrations were considered in the design. The beams of all bridges were precast and set into place by means of the cableway. The spillway deck is provided with expansion joints on sliding plates at selected points.

## Power House

The power house (see Fig. 4), is 340 feet in length, as required to house four units and to provide for a railroad entrance with adjoining working space. Below the level of the draft tube deck and railroad entrance, the substructure is, except for floors and partitions, of mass concrete reinforced to resist tailwater at El. 1348. Above this elevation, the walls, crane girders and roof are of reinforced concrete. Steel trusses are used to support the roof.

The service bay and railroad entrance are at the north end of the building, while the control bay is located at the south end on the upstream side. A 400 ton crane with 25 ton auxiliary hoist serves the generator room and service bay. A 30 ton gantry crane operates for the full length of the draft tube deck to handle the 21 foot by 18 foot draft tube gates. These gates are of welded structural steel and normally hang from latches at El. 1345.

The main transformers, the switchyard and the station service switchgear are located on the power house roof or on the rock shelf adjacent to the roof and at the same elevation. In order to bring the transformers to the roof level, a hatchway is located in the roof vertically over the railroad siding in the service bay. Through this hatchway, transformers and other lighter objects were hoisted by means of a 125 ton overhead traveling crane and set down at El. 1408. The runway for the crane is of a height and length to permit the transformers to be set on tracks near the hatchway, and then rolled to their permanent locations.

For the power house, 61,220 cu yd of concrete were required, including the encasement of two units.

## Turbines, Governors and Generators

Each of the two initial turbines has a rating of 120,000 hp at 210 foot net head and at 120 rpm. Runaway speed is about 1.85 times normal speed. Maximum discharge is about 5,700 cfs. The center line of the distributor is at El. 1300, 3 feet below normal tailwater, and 3 feet above minimum tailwater.

The runner is of cast steel in one piece, having an extreme diameter of 15 feet-9 inches. The spiral case is of welded steel with an entrance diameter of 17 feet-6 inches. The assembly of runner and 40 inch diameter shaft weighs 198,000 lb. The governor mechanism is a Woodward cabinet actuator, set to close the wicket gates in four seconds.

The generators are rated at 90,000 kva, 3 phase, 60 cycles, 14,400 volts, 80 per cent power factor and 120 rpm. They are of the umbrella vertical water wheel driven type, and are totally enclosed. Each rotor with shaft weighs 420 tons. The stator bore diameter is 357 inches.

Fins on the rotor of the generators force air through the stator coils and then through air coolers. This air is recirculated through the generator. A portion of the warm air is diverted through a louvered duct system for space heating, when required. A baffle seals off the generator from the turbine pit to avoid loss of carbon dioxide in event of the use of this gas for fire protection.

Power is transmitted over 69 kv lines to Trail, 13 miles away.

## Model Tests

A hydraulic model was made of the dam and tailrace. The model was constructed and tested at the Alden Hydraulic Laboratory of the Worcester Polytechnic Institute. It was built to a scale of 1:70, with velocities 1:8.37 of those of the prototype, and included the spillway, the downstream face of the power house, and about 500 feet of the channel downstream. Most of the tests were qualitative, in order to observe flow characteristics in the proposed forms of tailrace which were subjected to experiment. Fig. 7 shows the model in its final form.

The 30 foot high splitter wall on the apron was necessary because of the diversion program, having been used as part of a cofferdam to retain water first to the south, and finally, to the north, while plugging the sluices.

During model tests, it was discovered that stones would be drawn up from a deep hole in the channel, downstream of the spillway apron, by a reverse current, to strike the toe of the dam. Creation of the notch shown in the discharge apron between the power house and splitter wall bettered this condition by reducing the reverse current.

Since construction of a concrete spillway surface to tailwater level was not economically feasible for the spillway gates adjacent to the south bank, the model was constructed to permit the discharge to plunge over the brink of an inclined rock surface terminating about 100 feet above tailwater level. The spillway apron below was extended downstream to protect the rock from erosion and, for similar reasons, a protection wall was carried up to El. 1325 on the south bank. These gates will not be used unless an extreme flood should occur, which would be of short duration.

The main current of flood discharge water is along the south bank. This has a tendency to set up a clockwise eddy in the tailbay. Most of this tendency was found to be overcome by the water which was directed into the deep hole and which rose to the surface in the form of a boil immediately downstream of the power house. During the first few months of operation of the spillway, erosion of concrete was discovered on the upstream ends of the two vertical faces of the splitter wall, caused apparently by imperfect aeration under the jet as it breaks on the top of the wall.

The deep hole in the natural river bottom downstream of the dam at the south bank was filled with loose rock washed into it during excavation operations. As a consequence, spillway discharge from the prototype sets up the

FIG. 7- VIEW OF HYDRAULIC MODEL

clockwise eddy in the tailbay. This condition is expected to persist until flood conditions have removed the debris from the deep hole and from the tailrace channel farther downstream at which time, it is expected that the boil referred to will act to reduce the eddy.

## Preliminary Construction

Preliminary construction got under way on June 4, 1951. Before any major work on the project could be started, it was necessary to erect camp accommodations for personnel, and to provide shops, offices, warehouses, access roads and graded storage areas. The camp, which was opened in August of 1951, could accommodate over 500 men. A workshop and storage area were provided on the north bank of the Pend d'Oreille downstream of the dam site by utilizing excavated material from the dam and power house. Access to the south abutment area was provided by an existing logging road, and to the north abutment by a provincial road. Rail access to the site was excellent, a rail line actually passing through the area.

Ten miles of the highway from Trail, the nearest town and location of the principal smelters of Cominco, were adequate, but the remaining three miles were steep, narrow and winding, and had to be replaced by two miles of new highway along the east bank of the Columbia River. This road, entirely on fill, was protected by riprap obtained from the power house excavation. Over 400,000 cu yd of material were required. Construction of the road was accomplished between July and September of 1951.

## Construction Plant

Construction equipment used on the job was normal for the type of project.

A self-propelled and self-contained drilling rig, mounted on crawler treads, and capable of drilling holes of 6 inch diam, was used during the initial mass excavation stage in the power house and tailbay areas.

Larger items of equipment included a cableway of 1,770 foot span, having one fixed and one movable tower, and with a nominal capacity of 16 tons, which could serve the whole of the dam area and part of the power house. A 50 ton whirley crane located on a bench cut at the power house roof level served the power house area.

## Concrete

A comprehensive series of field tests of trial concrete mixes was conducted to determine the water/cement ratios required to meet the strength requirements of the specifications.

For air-entrained concrete made with 6 inch maximum size aggregate and Type I cement, the results of the tests in lb per sq in., together with water/cement ratios actually used for mix design purposes, were as follows:

| Nominal Design Strength | Actual Ultimate Strength | Req'd W/C Ratio as Indicated by Tests | W/C Ratio Actually Used on Job |
|---|---|---|---|
| 2,000 | 2,300 | 0.71 | 0.65 |
| 2,500 | 2,875 | 0.65 | 0.60 |
| 3,000 | 3,450 | 0.53 | 0.50 |
| 3,500 | 4,000 | 0.49 | 0.45 |

The lower values of allowable water/cement ratios were adopted to provide a cushion for the many variables normally encountered in concrete manufacture. Use of pozzolanic material, although considered, could not be justified economically for this job.

During early stages of construction prior to completion of the testing program and before results of tests on production concrete became available, minimum cement content of the various mixes was kept at conservative levels. As test results became known and inspection and operating personnel became more familiar with their work, the minimum cement content of mixes was progressively reduced. For 6 inch maximum concrete, for instance, the cement weight per cubic yard started at 347 lb, and was progressively reduced to 307 lb, 285 lb and finally 235 lb per cu yd. Compression test results for 96 sets of cylinders made from mass concrete containing 235 lb of cement per cu yd averaged 2,557 psi at 28 days. The average air content, after vibration, was 4.6 per cent.

## Placement of Concrete

Concrete in the dam was placed in 5 foot lifts, heavy 2-man air operated vibrators being used for compaction of the three 20 inch layers making up a lift. Laitance was removed from the surface of the concrete by use of the air-water jet method and, by careful timing, this gave excellent results. Three days were normally required between the completion of a lift and the start of the next lift.

Although Type I cement was used throughout, and no cooling of the concrete while mixing, or in the completed structure, was specified, contraction cracking in the dam has been noted only in the lower lifts of the block under the spillway regulating gate. These lifts were among the longest poured (measured parallel to flow) and were very narrow, being at levels in the foundation where block width (normal to flow) was restricted to a narrow rock ledge.

## Aggregate and Aggregate Production

Aggregates in the form of river-deposited sands and gravels are extremely plentiful in this section of the Columbia River valley. The deposits have been used for concrete for many years, with no evidence of failures attributable to the aggregates. An ample supply of aggregates was found in a large river flat on the east bank of the Columbia about 2 1/2 miles upstream of the job site, with material sufficiently clean to permit dry-screening of the coarse aggregates. Analyses of the materials from the test pits showed the area as a whole to be somewhat deficient in plus 3 inch sizes, and to contain a large surplus of sand. The sand was found to be deficient in the minus 48 mesh range, and to

contain the usual excessive amount of silt. Fortunately, large deposits of a fine clean sand containing a high proportion of the missing 48 mesh fraction were found to underlie this area.

The screening plant, with a capacity of 250 tons per hour, consisted of a primary jaw crusher, set to pass 6 inch minimum size, followed by vibratory screens of nominal 3 inch, 1 1/2 inch, 3/4 inch and No. 4 mesh openings to produce the four sizes of coarse aggregate specified. Material minus No. 4 mesh in size was wet-screened on a nominal No. 16 mesh screen, the material minus No. 16 mesh passing through a single screw sand classifier to remove the extreme fines and silt. A secondary gyrating cone type crusher, with a capacity of 18 tons per hour, was also provided, through which some of the material retained on the 3/4 inch screen could be recirculated to balance production of minus 3/8 inch material with indicated surpluses of the 3/4 inch to 1 1/2 inch fraction.

The shortage of plus 3 inch material was made up by high grading from selected areas of the pit. The shortage of No. 48 mesh sand was made up by the addition to the screening plant of a hopper and screw conveyor, discharging fine sand onto the sand screen.

From the stock piles at the gravel plant, aggregate was loaded by shovel onto trucks and hauled 1 1/2 miles to the intake hopper of a 30 inch conveyor belt system 3,500 feet long, and thence conveyed to stock piles and distributed by a traveling stacker over a reclaim tunnel located above the north abutment of the dam. Material from these piles was taken by belt conveyor to the batch plant at the north end of the dam, as required.

The batch plant contained hoppers for six aggregate sizes, cement storage bins, a 2 cu yd automatic scale batcher, and three 2 cu yd mixers discharging into a hopper. Mixed concrete was normally discharged into a 6 cu yd hopper mounted on a transfer car which, in turn, discharged either into the 6 cu yd cableway bucket for the dam, or into two 2 or 4 cu yd buckets placed on a car on an inclined rail system, from which they could be picked up by the whirley crane at the power house. Concrete could also be discharged into trucks.

Cement was delivered to the job site by rail in hopper cars, unloaded by screw and bucket conveyor into a 5,000 bbl storage silo at the siding, and transported to the batch plant by a compressed air pump and pipe system 2,400 feet long. Provision for unloading bulk cement from box cars was also provided in the form of a pneumatic unloader.

## River Diversion

The dam site is a hard rock ledge, forming a shelf over which the Pend d'Oreille flowed with a considerable velocity for a width of about 150 feet. Immediately downstream of the ledge was a rapids, of which the energy was spent in the deep pool downstream of the proposed discharge apron. Since this pool varied in depth from 40 to 100 feet, it was imperative to keep all cofferdam work within the shallower rapids area.

Since the north bank was the more accessible, a crib cofferdam was set on the river bottom, starting from the north bank. As shown in Fig. 6, Stage 1, the cofferdam was set at approximately midstream, taking into consideration the effect of widening the stream by excavating along the north bank. The cofferdam itself consisted of cribs, in plan 20 feet by 25 feet, framed to fit the river bottom. After being swung into place, the cribs were sunk by means of a rock fill. The cofferdam was carried downstream to a point near the end of the future spillway apron where the rock ledge stepped down. Since river

level beyond this point was 10 or 15 feet lower, a working space was obtained within the cofferdam without the need for a downstream wall.

The initial diversion was started in September of 1951 and proved to be slow and hazardous. The river bottom, being of clean solid rock, afforded no chance for driving piles for initial operations. Water velocities were from 10 to 20 fps.

In order to seal the cribbing, heavy timber sheathing was placed on the south, or water, face. This proved to be inadequate; therefore, a concrete wall, 16 feet thick, placed by the tremie method between forms, was constructed immediately inside the crib cofferdam. The construction of this wall provided the required protection for commencing foundation excavation. It was not possible to stop leakage under this wall because of the presence of rock seams, filled with loose material, located in the foundation area.

The construction schedule required completion of the dam by November of 1953 in order to begin generation of power by January 1, 1954, anticipating that the units would be ready by that date. The particular program to be followed subsequent to the unwatering of the initial foundation area was obviously dependent upon the time factor. Excavation within the cofferdam for the foundation was rushed during the period from January 1 to about April 1, 1952, but the delay caused by the necessity of constructing the concrete wall permitted only the completion of the south wall of the power house to El. 1348. This wall "closed in" the independent cofferdam required for carrying on work continuously within the power house area. The other objective of pouring the bases of two or three of the northerly spillway blocks was not accomplished because of the overtopping of the river cofferdam by the spring flood on April 24, 1952. Sufficient concrete had, however, been poured along the north bank in the cofferdam area to permit work to proceed continuously on the headworks blocks as soon as the spring flood of 1952 receded.

In spite of the failure to pour foundation concrete in this area, it was believed there was still a fighting chance to beat time and the river. A plan was formulated, accordingly, to divert the river through the excavated area, now appropriately called the north diversion channel. The upstream portion of the midstream cofferdam was removed and an upstream approach to the north diversion channel was blasted out. Rock removed from this area was dumped into the south channel. By August 22, most of the river flow had been diverted through the north channel. Final closure of the south channel was made on August 23, 1952, with a flow of 10,300 cfs, by blasting some 4,500 cu yd of rock from the steeply sloping south abutment. This material suddenly and effectually stopped the flow in the south channel.

By early September, 1952, the rock and earth dike had been completed at the upstream end of the south channel and a concrete wall constructed across the downstream end as shown in Stage 2 of Fig. 6. These, together with the midstream wall, formed the cofferdam within which the lower portions of the south river spillway blocks were poured by January 1, 1953.

With the completion of the two 20 foot by 30 foot sluices in these blocks (See Fig. 6, Stage 3) the upstream and downstream cofferdams were removed and diversion was begun through the sluices on January 25.

It was now possible to place cofferdams across the upper and lower ends of the north diversion channel and to work in the dry in this area, using the previously constructed blocks in the south diversion channel as a midstream cofferdam wall. Completion of the excavation in this area required removal of the midstream crib and 16 foot cofferdam wall, revealing the rock seams which previously caused leakage. Work on two of the three spillway blocks

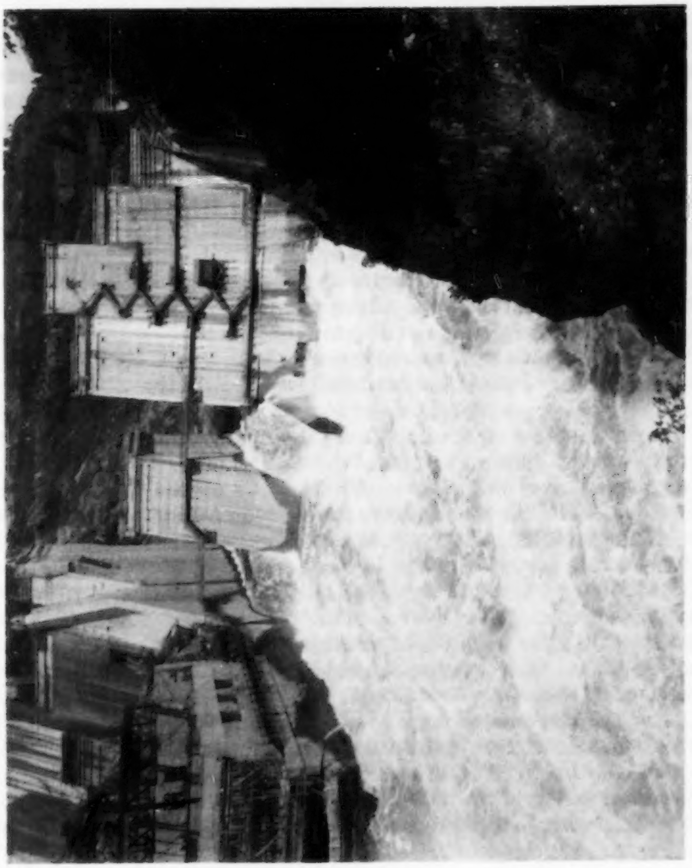

FIG. 8 - DIVERSION OF SPRING FLOOD OF 1953 THROUGH NOTCHES AND SLUICES.

in this area was stopped at El. 1340 to provide notches (See Fig. 6 & 8) to act as an emergency spillway for the spring flood of 1953. This spring flood exceeded 125,000 cfs, causing a head of 56 feet over the roofs of the sluices. By mid-July the flood receded to El. 1340, permitting resumption of work on the two spillway blocks which had been left at that elevation.

Before the end of 1953, dam construction, including equipment erection, was back on schedule, leaving plugging of the sluices to be accomplished early in 1954.

## Final Sluice Closure

Final closure of the dam was accomplished December 29, 1953, by sending down the four closure gates in front of the sluices. The two sluices were each 20 feet by 30 feet in cross section, and were each divided at their inlet ends, as shown in Fig. 5, to provide two entrances of 11 feet by 30 feet in order to reduce the size and weight of the gates to be handled.

The floor and sidewalls of the sluices were extended upstream from the face of the dam, to provide vertical gate slots and a sill. Steel sealing members were embedded in the lintel and the sides of each portal, while the sills were cushioned with hardwood, set flush with the floor. Structural steel guides extended above the slots to retain and guide the gates.

The gates, designed to resist the full hydrostatic head of 226 feet and believed to be among the largest ever used for this purpose, were fabricated from 36 WF beams framed horizontally, sealed with a thin steel skin plate on the upstream sides. Each gate was filled with concrete. The overall dimensions of the gates were 14 feet by 32 feet- 6 inches, inclusive of sealing timbers. The estimated weight of a filled gate was in excess of 110 tons, sufficient to assure gravity closure against the seal friction developed by a 60 foot differential head across the gate. While it was not anticipated that closure of the gates would have to be undertaken under such a head of water, it was considered advisable to provide for this contingency, which could occur in the case of a delay in the lowering of the last one or two of the four gates.

It was obvious that a gate of such weight could not be dropped without control for any appreciable distance, even under water, without danger of damage to itself and the sill. The problem of providing some means of controlling the rate of descent of the gates was the subject of much study. The principle of the hydraulic shock absorber, or dashpot, was suggested, and met with general approval. For this purpose, each gate was suspended by means of a 2 3/8 inch wire rope in its guide, 31 feet above the sluice floor, where it was filled with concrete. To form the "hydraulic brake", as it was called, a 24 inch diam steel pipe was embedded in the upstream overhang of the spillway crest, and located concentric with each rope just below the rope support beam at the crest. See Fig. 5. The upper rope socket was attached to a 12 inch by 1 inch bar bent over the support beam.

Each dashpot piston was composed of a thick circular steel plate with tapered hole to bear against the tapered basket of the upper rope socket. The lower end of each 24 inch cylinder was welded to a steel base plate. Below this was attached a sleeve and packing gland.

Precautions were taken to avoid the water hammer which would occur if the suddenly released gate load were immediately transferred to a confined body of water below the piston. It was also considered important to avoid impact in the rope if its full tension were momentarily partially relieved and then suddenly restored. Accordingly, each 24 inch pipe or cylinder was split

and rewelded at its upper end to form a cone, in which the piston, as it descended, would meet a gradually increasing, rather than a suddenly applied, water resistance. Assuming a maximum terminal velocity of 15 fps to be satisfactory, the wall clearance of the piston was made 3/16 inch. Theoretical hydrostatic pressure was 450 psi under full gate weight. Release of the gates was accomplished by flame cutting each 12 inch by 1 inch bent bar where it passed over its support beam. To make each "hydraulic brake" operative, it was filled with water, with a constant supply to make up for leakage. The south gate was released first, as a trial, to permit adjustments in procedure to be made, if found desirable. Burning was carried out with a torch, inserted through a slot cut in an inclined steel plate erected over the top of each cylinder to provide protection for the operator against the fountain of water forced from the cylinder. The bar was cut toward the center from each edge to avoid extreme eccentricities at rupture.

Release of the first gate was without incident, the gate descending smoothly in approximately 4 seconds, through a 27 foot depth of water. All displaced water escaped either through the lower gland or past the descending piston. The remaining gates were then simultaneously and successfully released. Subsequent inspection of the gates from inside the sluices showed one gate to be entirely tight, two to have minor leaks past the seals, and one with considerable leakage at a lower corner. Cinders dropped upstream of the gates after filling of the reservoir reduced the leaks to an amount easily handled by the installed drains.

While reservoir overflow was being passed over the north end of the spillway, both sluices were filled with concrete and, later, sealed by pressure grouting.

## Power House and Penstock Excavation

Completion of the south wall of the power house to above flood stage in April 1952 allowed excavation for the power house to proceed without interruption.

Excavation methods were not unusual, except for the employment in the mass excavation stages of 70 foot deep, 6 inch diam drill holes, spaced on 20 foot centers. Loading was based on a factor of 0.75 lb per cu yd. Wagon drills and jackhammers were used for trimming to line and grade.

Excavation of the first of the four penstock tunnels started in March 1952, and all were completed by mid-July 1952, 15,000 cu yd of rock being removed. The tunnels were driven full size from the upper end for approximately 20 feet, and thereafter a pilot shaft was driven to connect with full size headings started from the power house. The pilot shafts then served as muck chutes as the tunnels were brought out to full size from the top down. Timbering was required only near the upper ends of the tunnels.

Power house excavation was substantially completed, apart from the removal of the rock barrier adjacent to the river, in September, 1952. Including rock removed at a later date, approximately 291,000 cu yd of rock were excavated in the dry for the power house and substation and to provide permanent rail and road access to the area.

Removal of the 35,000 cu yd of rock, comprising the cofferdam barrier at the power house, was first attempted October 9, 1953. Horizontal blast holes, drilled from inside the cofferdam all over the barrier face, were loaded and blasted in the dry using delays, in order that the power house, close by, would be subjected to a series of small shocks rather than one severe one. The

blast was only partly successful because a large portion of the dynamite in the lower and outer portion of the barrier had been immersed for three weeks in drill holes containing running water, and failed to explode.

Construction of a concrete tremie cofferdam to close the gap in the remains of the barrier made possible the unwatering of the cofferdam area. It was then decided to redrill the deeper section of the barrier farthest from shore by using vertical diamond drill holes. This section was reblasted December 23, 1953, with the cofferdam area flooded. Four of the six installed draft tube gates were heavily damaged despite various precautionary measures.

The remainder of the barrier was drilled and shot in small sections subsequent to the date of closing the sluices. By stopping all flow at the dam for periods of a few hours, it was possible to expedite the drilling by draining the tailwater pool to the Columbia River.

### Grout Curtain

The rock on which the dam is founded consists chiefly of highly metamorphosed argillite and fine grained agglomerate, intersected by a large number of lamprophyre and diorite dikes. The area is intersected by numerous minor faults and fracture planes, sealed by calcite which has been removed by weathering to depths as great as 50 feet.

The purpose of the grout program was the resealing of the weathered calcite veins, and was accomplished by drilling two series of holes near the axis of the dam. The first and upstream line was, in general, 25 feet deep, drilled on 10 foot centers. On completion of drilling and grouting these primary holes, a second line 10 feet farther downstream, with holes centered between the primary holes, was drilled to depths varying between 50 and 75 feet, depending upon inspection of cores and results of water testing of 100 foot test holes drilled under each block.

Completion of this program involved the drilling of 12,600 feet of diamond drill holes and 2,900 feet of wagon drill holes. Grout consumption was 2,100 bags.

### Start-up

Although the first unit was run for a few hours on January 14, 1954, the dry out run could not be started until February 18, when the last of the damaged draft tube gates had been removed. The first unit was turned over to Cominco on March 15, 1954 and the second unit went into service on May 18, 1954.

Fig. 9 is a downstream view of the completed project passing the spring flood of 1954. The arrangement of gates shown in operation is about as predicted by model tests to result in the most favorable hydraulic conditions downstream of the dam and power house for a flood of this magnitude.

## ACKNOWLEDGMENTS

The Consolidated Mining and Smelting Company of Canada Limited, owner, was represented by Edwin M. Stiles, Consulting Engineer of that company.

Concrete and excavation work on the dam and power house were under the direction of Simond Piedmont, General Superintendent of Northern Construction Company of Vancouver, B. C.

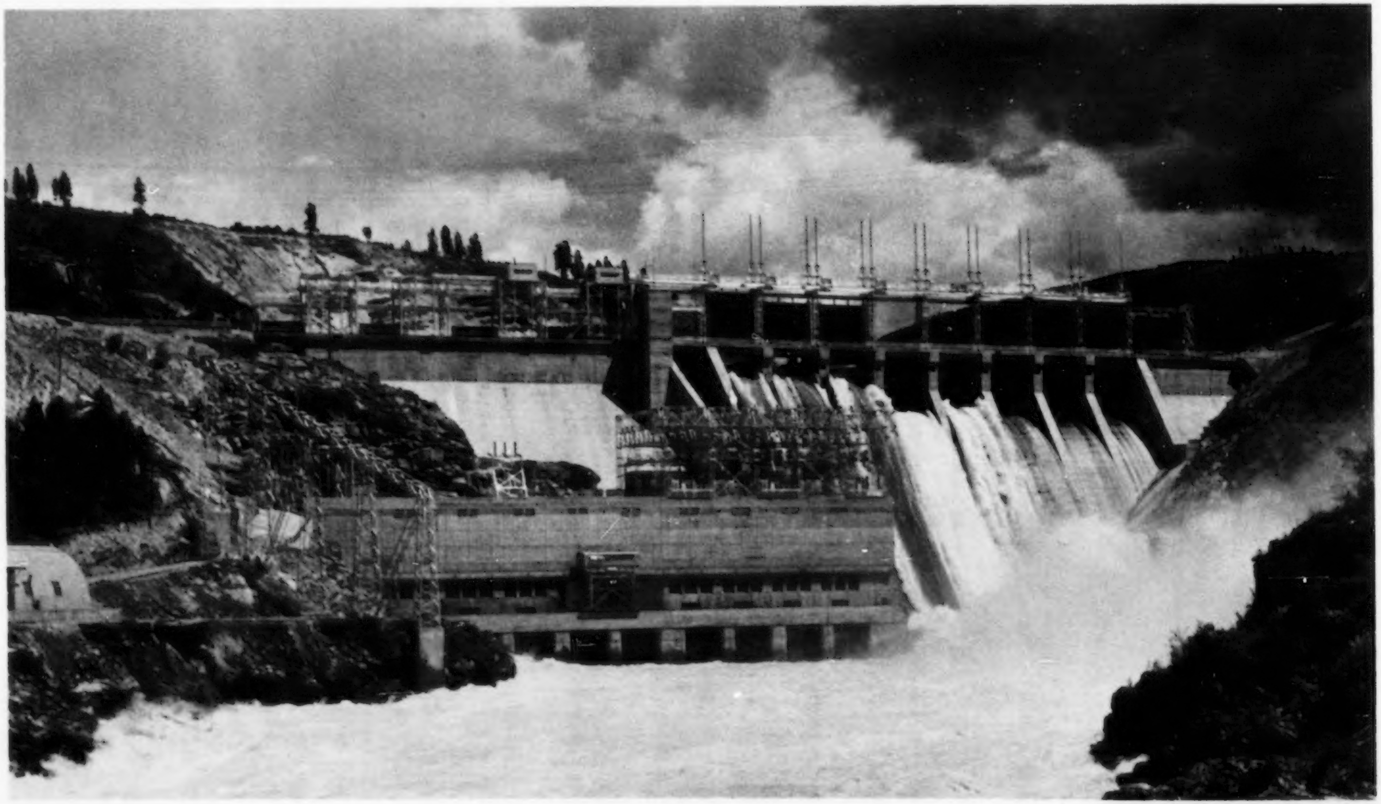

FIG. 9 - THE COMPLETED WANETA PROJECT DURING THE SPRING FLOOD OF 1954.

Construction of the entire project was in charge of Paul Brown, Project Manager for Stone & Webster Canada Limited. Project planning and design were under the direction of George R. Strandberg, M., ASCE, then Chief Hydraulic Engineer for the Stone & Webster Engineering Corporation.

Prof. Leslie J. Hooper, M., ASCE, Director of the Alden Hydraulic Laboratory, supervised the construction and testing of the hydraulic models.

## PROCEEDINGS-SEPARATES

The technical papers published in the past year are presented below. Technical-division sponsorship is indicated by an abbreviation at the end of each Separate Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways (WW) divisions. For titles and order coupons, refer to the appropriate issue of "Civil Engineering" or write for a cumulative price list.

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c. Discussion of several papers, grouped by Divisions.
d. Presented at the Atlanta (Ga.) Convention of the Society in February, 1954.
e. Presented at the Atlantic City (N.J.) Convention in June, 1954.